Chapter 6
Bearing Capacity

6-1. Scope

This chapter provides guidance for the determination of the ultimate and allowable bearing stress values for foundations on rock. The chapter is subdivided into four sections with the following general topic areas: modes and examples of bearing capacity failures; methods for computing bearing capacity; allowable bearing capacity; and treatment methods for improving bearing capacity.

6-2. Applicability

a. Modes of failure, methods for estimating the ultimate and allowable bearing capacity, and treatments for improving bearing capacity are applicable to structures founded directly on rock or shallow foundations on rock with depths of embedments less than four times the foundation width. Deep foundations such as piles, piers, and caissons are not addressed.

b. As a rule, the final foundation design is controlled by considerations such as deformation/settlement, sliding stability or overturning rather than by bearing capacity. Nevertheless, the exceptions to the rule, as well as prudent design, require that the bearing capacity be evaluated.

Section I
Failure Modes

6-3. General

Bearing capacity failures of structures founded on rock masses are dependent upon joint spacing with respect to foundation width, joint orientation, joint condition (open or closed), and rock type. Figure 6-1 illustrates typical failure modes according to rock mass conditions as modified from suggested modes by Sowers (1979) and Kulhawy and Goodman (1980). Prototype failure modes may actually consist of a combination of modes. For convenience of discussion, failure modes will be described according to four general rock mass conditions: intact, jointed, layered, and fractured.

6-4. Intact Rock Mass

For the purpose of bearing capacity failures, intact rock refers to a rock mass with typical discontinuity spacing (S term in Figure 6-1) greater than four to five times the width (B term in Figure 6-1) of the foundation. As a rule, joints are so widely spaced that joint orientation and condition are of little importance. Two types of failure modes are possible depending on rock type. The two modes are local shear failure and general wedge failure associated with brittle and ductile rock, respectively.

a. Brittle rock. A typical local shear failure is initiated at the edge of the foundation as localized crushing (particularly at edges of rigid foundations) and develops into patterns of wedges and slip surfaces. The slip surfaces do not reach the ground surface, however, ending somewhere in the rock mass. Localized shear failures are generally associated with brittle rock that exhibit significant post-peak strength loss (Figure 6-1a).

b. Ductile rock. General shear failures are also initiated at the foundation edge, but the slip surfaces develop into well defined wedges which extend to the ground surface. General shear failures are typically associated with ductile rocks which demonstrate post-peak strength yield (Figure 6-1b).

6-5. Jointed Rock Mass

Bearing capacity failures in jointed rock masses are dependent on discontinuity spacing, orientation, and condition.

a. Steeply dipping and closely spaced joints. Two types of bearing capacity failure modes are possible for structures founded on rock masses in which the predominant discontinuities are steeply dipping and closely spaced as illustrated in Figure 6-1c and 6-1d. Discontinuities that are open (Figure 6-1c) offer little lateral restraint. Hence, failure is initiated by the compressive failure of individual rock columns. Tightly closed discontinuities (Figure 6-1d) on the other hand, provide lateral restraint. In such cases, general shear is the likely mode of failure.

b. Steeply dipping and widely spaced joints. Bearing capacity failures for rock masses with steeply dipping joints and with joint spacing greater than the width of the foundation (Figure 6-1e) are likely to be initiated by splitting that eventually progresses to the general shear mode.

c. Dipping joints. The failure mode for a rock mass with joints dipping between 20 to 70 degrees with respect to the foundation plane is likely to be general shear (Figure 6-1f). Furthermore, since the discontinuity

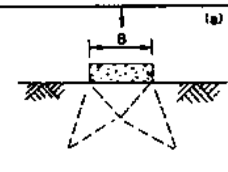
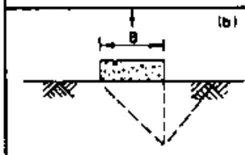
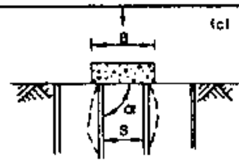
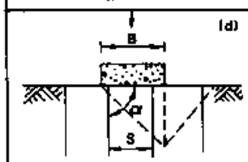
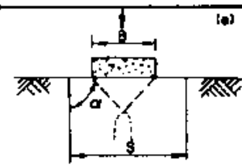
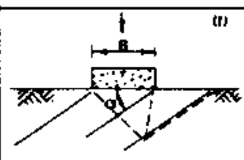
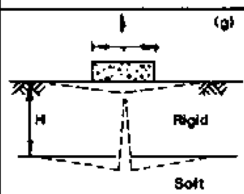
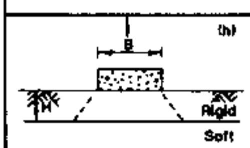
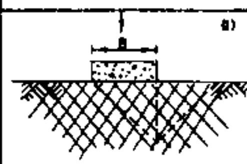
Rock Mass Conditions			Failure		Bearing Capacity Equation No.
	Joint Dip	Joint Spacing	Illustration	Mode	
INTACT	N/A	S>>B		Brittle Rock: Local shear failure caused by localized brittle fracture.	Eq. 6.4
				Ductile Rock: General shear failure along well defined failure surfaces.	Eq. 6.1
STEEPLY DIPPING JOINTS	70°< α >90°	S<B		Open Joints: Compressive failure of individual rock columns. Near vertical joints set(s)	Eq. 6.5
				Closed Joints: General shear failure along well defined failure surfaces. Near vertical joint(s).	Eq. 6.1
		S>B		Open or Closed Joints: Failure initiated by splitting leading to general shear failure. Near vertical joint set(s).	Eq. 6.6
JOINTED	20°< α <70°	S<B or S>B if failure wedge can develop along joints		General shear failure with potential for failure along joints. Moderately dipping joint set(s).	Eq. 6.3
LAYERED	0< α <20	Limiting Values of H with respect to B is dependent upon material properties		Thin Rigid Upper Layer: Failure is iniated by tensile failure caused by flexure of thin rigid upper layer.	N/A
				Thin Upper Rigid Layer: Failure is initiated by punching tinsile failure of the thin rigid upper layer.	N/A
FRACTURED	N/A	S<<B		General shear failure with irregular failure surface through rock mass. Two or more closely spaced joint sets.	Eq. 6.3

Figure 6-1. Typical bearing capacity failure modes associated with various rock mass conditions

represents major planes of weakness, a favorably oriented discontinuity is likely to define at least one surface of the potential shear wedge.

6-6. Layered Rock Mass

Failure modes of multilayered rock masses, with each layer characterized by different material properties, are complicated. Failure modes for two special cases, however, have been identified (Sowers 1979). In both cases the founding layer consists of a rigid rock underlain by a soft highly deformable layer, with bedding planes dipping at less than 20 degrees with respect to the foundation plane. In the first case (Figure 6-1g), a thick rigid layer overlies the soft layer, while in the second case (Figure 6-1h) the rigid layer is thin. In both cases, failure is initiated by tensile failure. However, in the first case, tensile failure is caused by flexure of the rigid thick layer, while in the second case, tensile failure is caused by punching through the thin rigid upper layer. The limiting thickness of the rigid layer in both cases is controlled by the material properties of each layer.

6-7. Highly Fractured Rock Masses

A highly fractured rock mass is one that contains two or more discontinuity sets with typical joint spacings that are small with respect to the foundation width (Figure 6-1i). Highly fractured rock behaves in a manner similar to dense cohesionless sands and gravels. As such, the mode of failure is likely to be general shear.

6-8. Secondary Causes of Failure

In addition to the failure of the foundation rock, aggressive reactions within the rock mineralogy or with ground water or surface water chemistry can lead to bearing capacity failure. Examples include: loss of strength with time typical of some clay shales; reduction of load bearing cross-section caused by chemical reaction between the foundation element and the ground water or surface water; solution-susceptible rock materials; and additional stresses imposed by swelling minerals. Potential secondary causes should be identified during the site investigation phase of the project. Once the potential causes have been identified and addressed, their effects can be minimized.

Section II
Methods for Computing Bearing Capacity

6-9. General

There are a number of techniques available for estimating the bearing capacity of rock foundations. These techniques include analytical methods, traditional bearing capacity equations, and field load tests. Of the various methods, field load tests are the least commonly used for two reasons. First, as discussed in Chapter 4, field load tests, such as the plate bearing test, are expensive. Second, although the test provides information as to the load that will cause failure, there still remains the question of scale effects.

6-10. Definitions

Two terms used in the following discussions require definition. They are the ultimate bearing capacity and allowable bearing value. Definition of the terms are according to the American Society for Testing and Materials.

a. Ultimate bearing capacity. The ultimate bearing capacity is defined as the average load per unit area required to produce failure by rupture of a supporting soil or rock mass.

b. Allowable bearing capacity value. The allowable bearing capacity value is defined as the maximum pressure that can be permitted on a foundation soil (rock mass), giving consideration to all pertinent factors, with adequate safety against rupture of the soil mass (rock mass) or movement of the foundation of such magnitude that the structure is impaired. Allowable bearing values will be discussed in Section III of this chapter.

6-11. Analytical Methods

The ultimate bearing capacity may be implicitly estimated from a number of analytical methods. The more convenient of these methods include the finite element and limit equilibrium methods.

a. Finite element method. The finite element method is particularly suited to analyze foundations with unusual shapes and/or unusual loading conditions as well as in situations where the foundation rock is highly variable. For example, the potential failure modes for the layered foundation rock cases illustrated in Figures 6-1g and 6-1h will require consideration of the interactions between the soft and rigid rock layers as well as between the rigid rock layer and the foundation. The primary disadvantage of the finite element method is that the method does not provide a direct solution for the ultimate bearing capacity. Such solutions require an analyses of the resulting stress distributions with respect to a suitable failure criterion. In addition to the method's ability to address complex conditions, the primary advantage is that the method provides direct solutions for deformation/settlement.

b. Limit equilibrium. The limit equilibrium method is applicable to bearing capacity failures defined by general wedge type shear, such as illustrated in Figures 6-1b, 6-1d, 6-1f, and 6-1i. The limit equilibrium method, as applied to sliding stability, is discussed in Chapter 7. Although the principals are the same as in sliding stability solutions, the general form of the equations presented in Chapter 7 needs to be cast in a form compatible with bearing capacity problems. The ultimate bearing capacity corresponds to the foundation loading condition necessary to cause an impending state of failure (i.e. the loading case where the factor of safety is unity).

6-12. Bearing Capacity Equations

A number of bearing capacity equations are reported in the literature which provide explicit solutions for the ultimate bearing capacity. As a rule, the equations represent either empirical or semi-empirical approximations of the ultimate bearing capacity and are dependent on the mode of potential failure as well as, to some extent, material properties. In this respect, selection of an appropriate equation must anticipate likely modes of potential failure. The equations recommended in the following discussions are presented according to potential modes of failure. The appropriate equation number for each mode of failure is given in Figure 6-1.

a. General shear failure. The ultimate bearing capacity for the general shear mode of failure can be estimated from the traditional Buisman-Terzaghi (Terzaghi 1943) bearing capacity expression as defined by Equation 6-1. Equation 6-1 is valid for long continuous foundations with length to width ratios in excess of ten.

$$q_{ult} = cN_c + 0.5\ \gamma B N_\gamma + \gamma D N_q \qquad (6\text{-}12)$$

where

q_{ult} = the ultimate bearing capacity

γ = effective unit weight (i.e. submerged unit wt. if below water table) of the rock mass

B = width of foundation

D = depth of foundation below ground surface

c = the cohesion intercepts for the rock mass

The terms N_c, N_γ, and N_q are bearing capacity factors given by the following equations.

$$N_c = 2\ N\phi^{1/2}\ (N\phi + 1) \qquad (6\text{-}2a)$$

$$N\gamma = N\phi^{1/2} \left(N_\phi^2 - 1\right) \qquad (6\text{-}2b)$$

$$N_q = N_\phi^2 \qquad (6\text{-}2c)$$

$$N_\phi = \tan^2\ (45 + \phi/2) \qquad (6\text{-}2d)$$

where

ϕ = angle of internal friction for the rock mass

Equation 6-1 is applicable to failure modes in which both cohesion and frictional shear strength parameters are developed. As such, Equation 6-1 is applicable to failure modes illustrated in Figures 6-1b and 6-1d.

b. General shear failure without cohesion. In cases where the shear failure is likely to develop along planes of discontinuity or through highly fractured rock masses such as illustrated in Figures 6-1f and 6-1i, cohesion cannot be relied upon to provide resistance to failure. In such cases the ultimate bearing capacity can be estimated from the following equation:

$$q_{ult} = 0.5\ \gamma B N_\gamma + \gamma D N_q \qquad (6\text{-}3)$$

All terms are as previously defined.

c. Local shear failure. Local shear failure represents a special case where failure surfaces start to develop but do not propagate to the surface as illustrated in Figure 6-1a. In this respect, the depth of embedment contributes little to the total bearing capacity stability. An expression for the ultimate bearing capacity applicable to localized shear failure can be written as:

$$q_{ult} = cN_c + 0.5\gamma B N_\gamma \tag{6-4}$$

All terms are as previously defined.

d. Correction factors. Equations 6-1, 6-3, and 6-4 are applicable to long continuous foundations with length to width ratios (L/B) greater than ten. Table 6-1 provides correction factors for circular and square foundations, as well as rectangular foundations with L/B ratios less than ten. The ultimate bearing capacity is estimated from the appropriate equation by multiplying the correction factor by the value of the corresponding bearing capacity factor.

Table 6-1
Correction factors (after Sowers 1979)

Foundation Shape	C_c N_c Correction	$C\gamma$ N_γ Correction
Circular	1.2	0.70
Square	1.25	0.85
Rectangular		
L/B = 2	1.12	0.90
L/B = 5	1.05	0.95
L/B = 10	1.00	1.00

Correction factors for rectangular foundations with L/B ratios other than 2 or 5 can be estimated by linear interpolation.

e. Compressive failure. Figure 6-1c illustrates a case characterized by poorly constrained columns of intact rock. The failure mode in this case is similar to unconfined compression failure. The ultimate bearing capacity may be estimated from Equation 6-5.

$$q_{qult} = 2\ c\ \tan\ (45 + \phi/2) \tag{6-5}$$

All parameters are as previously defined.

f. Splitting failure. For widely spaced and vertically oriented discontinuities, failure generally initiates by splitting beneath the foundation as illustrated in Figure 6-1e. In such cases Bishnoi (1968) suggested the following solutions for the ultimate bearing capacity:

For circular foundations

$$q_{ult} = JcN_{cr} \tag{6-6a}$$

For square foundations

$$q = 0.85\,JcN_{cr} \tag{6-6b}$$

For continuous strip foundations for L/B ≤ 32

$$q_{ult} = JcN_{cr}/(2.2 + 0.18\ L/B) \tag{6-6c}$$

where

J = correction factor dependent upon thickness of the foundation rock and width of foundation.

L = length of the foundation

The bearing capacity factor N_{cr} is given by:

$$N_{cr} = \frac{2N_\phi^2}{1+N_\phi} \quad (\cot\phi)\,(S/B) \left(1 - \frac{1}{N_\phi}\right) - N\phi\ (\cot\phi) + 2N\phi^{1/2} \tag{6-6d}$$

All other terms are as previously defined. Graphical solutions for the correction factor (J) and the bearing capacity factor (N_{cr}) are provided in Figures 6-2 and 6-3, respectively.

g. Input parameters. The bearing capacity equations discussed above were developed from considerations of the Mohr-Columb failure criteria. In this respect, material property input parameters are limited to two parameters; the cohesion intercept (c) and the angle of internal friction (ϕ). Guidance for selecting design shear strength parameters is provided in Chapter 4. However, since rock masses generally provide generous margins of safety against bearing capacity failure, it is recommended that

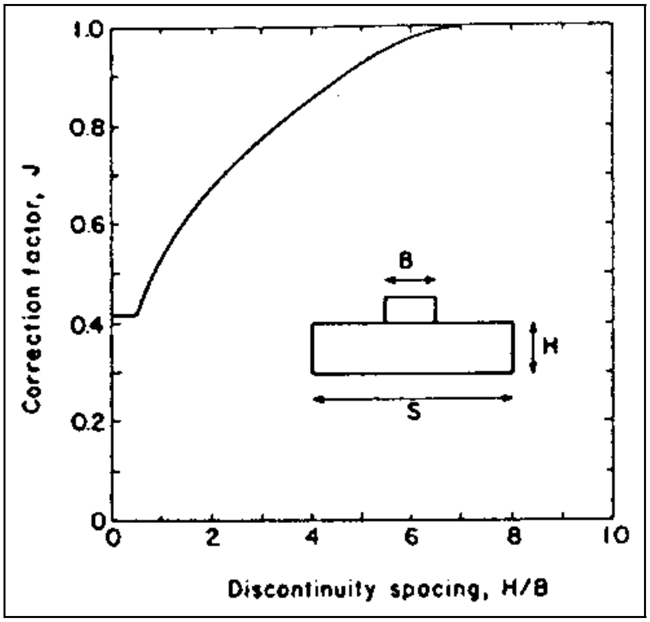


Figure 6-2. Correction factor for discontinuity spacing with depth (after Bishnoi 1968)

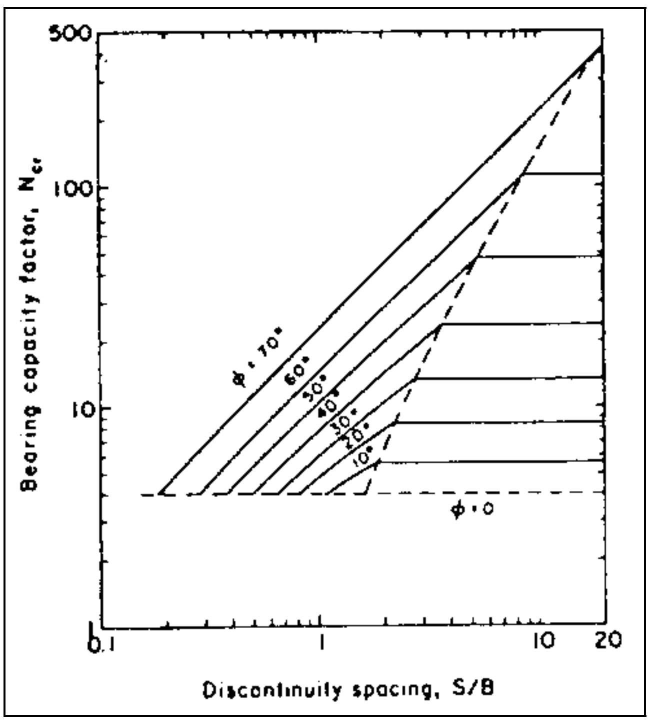


Figure 6-3. Bearing capacity factor for discontinuity spacing (after Bishnoi 1968)

initial values of c and ϕ selected for assessing bearing capacity be based on lower bound estimates. While inexpensive techniques are available on which to base lower bound estimates of the friction angle, no inexpensive techniques are available for estimating lower bound cohesion values applicable to rock masses. Therefore, for computing the ultimate bearing capacity of a rock mass, the lower bound value of cohesion may be estimated from the following equation.

$$c = \frac{q_u \quad (s)}{2 \tan \left(45 + \frac{\phi}{2}\right)} \tag{6-7a}$$

where

q_u = unconfined compressive strength of the intact rock from laboratory tests.

$$s = \exp \frac{(RMR - 100)}{9} \tag{6-7b}$$

All other parameters are as previously defined.

6-13. Eccentric Load on a Horizontal Foundation

Eccentric loads acting on foundations effectively reduce the bearing capacity. Figure 6-4a illustrates a typical structure subjected to an eccentric load. In order to prevent loss of rock/structure contact at the minimum stress edge of the foundation (Figure 6-4a), the structure must be designed so that the resultant of all forces acting on the foundations passes through the center one-third of the foundation. As indicated in Figure 6-4a, the stress distribution can be approximated by linear relationship. Equations 6-8a and 6-8b define the approximate maximum and minimum stress, respectively.

$$q_{(max)} = \frac{Q}{B}\left(1 + \frac{6e}{B}\right) \tag{6-8a}$$

$$q_{(min)} = \frac{Q}{B}\left(1 - \frac{6e}{B}\right) \tag{6-8b}$$

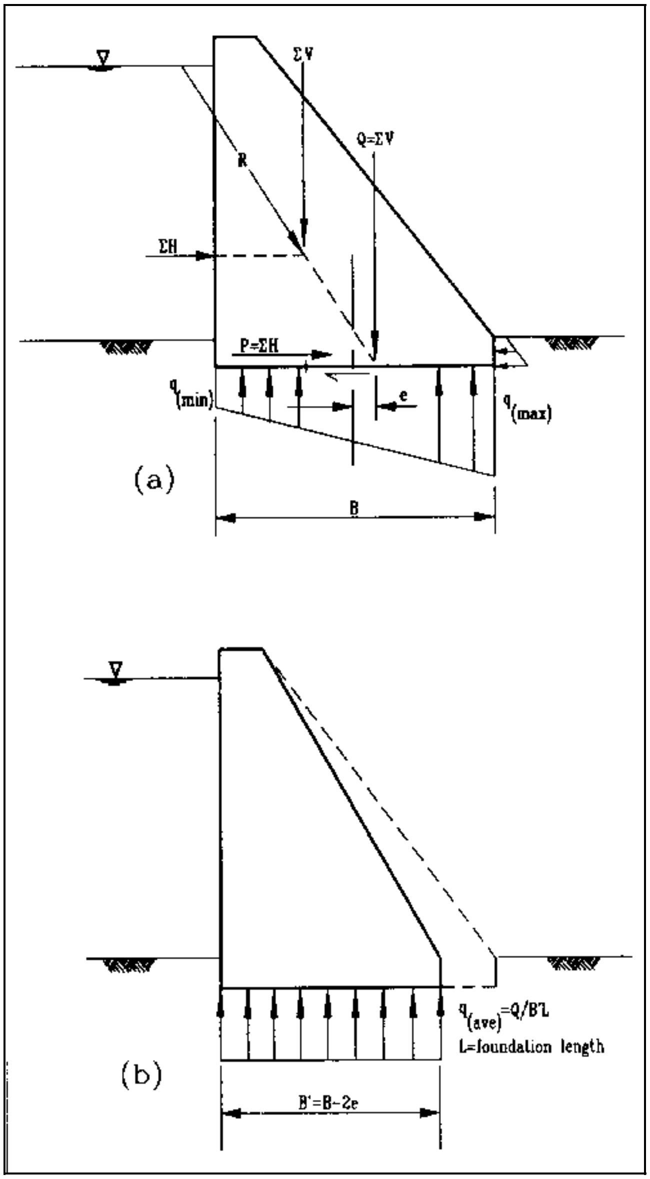


Figure 6-4. Typical eccentrically loaded structure foundation

where

$q_{(max)}$ = maximum stress

$q_{(min)}$ = minimum stress

Q = vertical force component of the resultant of all forces acting on the structure

B = the foundation width

e = distance from the center of the foundation to the vertical force component Q

The ultimate bearing capacity of the foundation can be approximated by assuming that the vertical force component Q is uniformly distributed across a reduced effective foundation width as indicated in Figure 6-4b. The effective width is defined by the following equation.

$$B' = B - 2e \tag{6-10}$$

The effective width (B') is used in the appropriate bearing capacity equation to calculate the ultimate bearing capacity.

6-14. Special Design Cases

The bearing capacity equations discussed above are applicable to uniformly loaded foundations situated on planar surfaces. Frequently, designs suited to the particular requirements of a project require special considerations. Special design cases for which solutions of the ultimate bearing capacity are readily available are summarized in Figure 6-5. As indicated in Figure 6-5, these special cases include inclined loads, inclined foundations, and foundations along or near slopes. Guidance for these special cases is provided in EM 1110-2-2502 and the NAVDOCKS DM-7. Ultimate bearing capacity solutions for special design cases should be in keeping with the modes of failure summarized in Figure 6-1.

Section III
Allowable Bearing Capacity Value

6-15. General

The allowable bearing capacity value is defined in paragraph 6-10*b*. In essence, the allowable bearing capacity is the maximum limit of bearing stress that is allowed to be applied to the foundation rock. This limiting value is intended to provide a sufficient margin of safety with respect to bearing failures and deformation/settlement. Nevertheless, a prudent design dictates that, once the allowable bearing capacity value has been determined, a separate calculation be performed in order to verify that

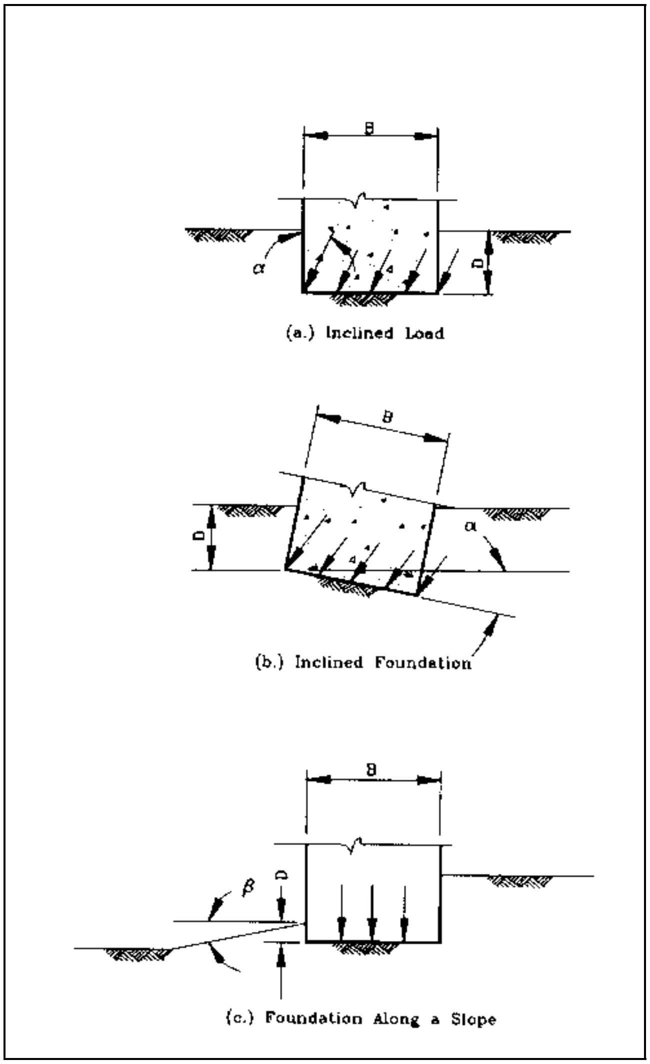


Figure 6-5. Special foundation design cases

the allowable differential deformation/settlement is not exceeded.

6-16. Determination

There are at least three approaches for determining allowable bearing capacity values. First, the allowable value may be determined by applying a suitable factor of safety to the calculated ultimate bearing capacity. The selection of final allowable bearing values used in design of hydraulic structures must be based on the factor of safety approach in which all site specific conditions and unique problems of such structures are considered. Second, allowable values may be obtained from various building codes. However, building codes, in general, apply only to residential or commercial buildings and are not applicable to the unique problems of hydraulic structures. Finally, allowable values may be obtained from empirical correlations. As a rule, empirical correlations are not site specific and hence should be used only for preliminary design and/or site evaluation purposes. Regardless of the approach used, the allowable value selected for final design must not exceed the value obtained from the factor of safety considerations discussed in paragraph 6-16*a*.

a. Factor of safety. The allowable bearing capacity value, q_a, based on the strength of the rock mass is defined as the ultimate bearing capacity, q_{ult}, divided by a factor of safety (FS):

$$q_a = q_{ult}/FS \tag{6-11}$$

The average stress acting on the foundation material must be equal to or less than the allowable bearing capacity according to the following equation.

$$Q/BL \leq q_a \tag{6-12}$$

For eccentrically loaded foundations the B' value (i.e. Equation 6-10) is substituted for the B term in Equation 6-12. The factor of safety considers the variability of the structural loads applied to the rock mass, the reliability with which foundation conditions have been determined, and the variability of the potential failure mode. For bearing capacity problems of a rock mass, the latter two considerations are the controlling factors. For most structural foundations, the minimum acceptable factor of safety is 3 with a structural load comprised of the full dead load plus the full live load.

b. Building codes. Allowable bearing capacity values that consider both strength and deformation/settlement are prescribed in local and national building codes. Local codes are likely to include experience and geology within their jurisdiction while national codes are more generic. For example, a local code will likely specify a particular rock formation such as "well-cemented Dakota sandstone" while a national code may use general terminology such as "sedimentary rock in sound condition." As a rule, allowable values recommended by the building codes are conservative.

c. Empirical correlations. Peck, Hanson, and Thornburn (1974) suggested an empirical correlation between the allowable bearing capacity stress and the RQD, as shown in Figure 6-6. The correlation is intended

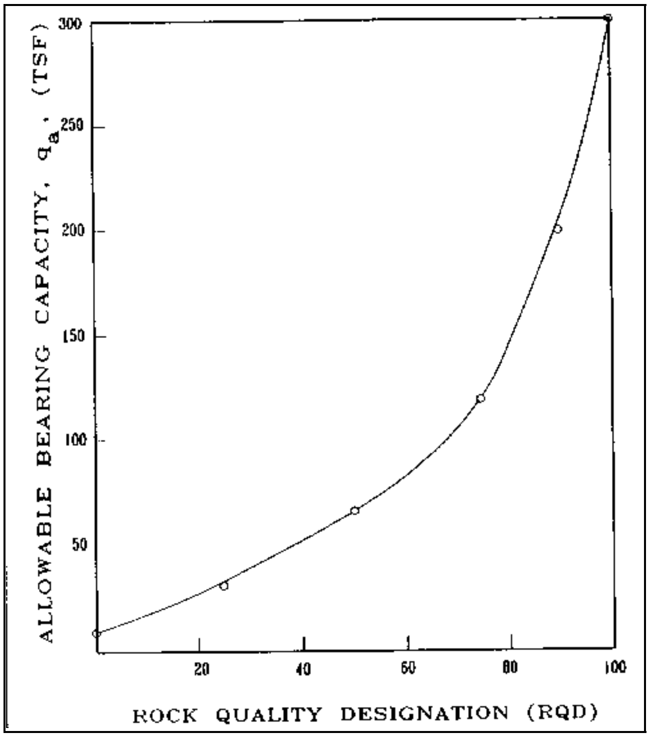


Figure 6-6. Allowable contact pressure on jointed rock

for a rock mass with discontinuities that "are tight or are not open wider than a fraction of an inch."

6-17. Structural Limitations

The maximum load that can be applied to a rock foundation is limited by either the rock's ability to sustain the force without failure or excessive settlement, or the ability of the substructure to sustain the load without failure or excessive deformation. In some cases the structural design of the foundation element will dictate the minimum element size, and, consequently, the maximum contact stress on the rock. For typical concrete strengths in use today, the strength of the concrete member is significantly less than the bearing capacity of many rock masses.

Section IV
Treatment Methods

6-18. General

Treatment methods for satisfying bearing capacity requirements are essentially the same as those for satisfying deformation/settlement requirements discussed in Chapter 5. In addition to the previously discussed methods, an examination of the general ultimate bearing capacity equation (i.e. Equation 6-1) indicates the importance of two parameters not directly related to deformability. These two parameters are the effective unit weight of the foundation rock and the depth of the foundation below the ground surface.

6-19. Effective Unit Weight

For foundations below the water table the effective unit weight is the unit weight of the foundation rock minus the unit weight of water (i.e. submerged unit weight of the rock). Hence, foundations located above the water table will develop significantly more resistance to potential bearing capacity failures than foundations below the water table.

6-20. Foundation Depth

Foundations constructed at greater depths may increase the ultimate bearing capacity of the foundation. The improved capacity is due to a greater passive resisting force and a general increase in rock mass strength with depth. The increased lithostatic pressure closes discontinuities, and the rock mass is less susceptible to surficial weathering. Occasionally, deeper burial may not be advantageous. A region with layers of differing rock types may contain weaker rock at depth. In such an instance, a strong rock might overlie a layer such as mudstone, or, if in a volcanic geology, it might be underlain by a tuff or ash layer. In these instances, deeper burial may even decrease the bearing capacity. The geologic investigation will determine this possibility.